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Geodatabases to improve geotechnical design and modelling

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Abstract: Geotechnical designers and modellers must capture and quantify the variability of key soil properties to make engineering decisions. There is a long history in geotechnical engineering of assembling large databases of past soil tests. This paper shows the use of geotechnical databases in two contexts: (a) slope stability modelling in the Eastern Caribbean and (b) settlement response of bored piles in London Clay.

Keywords: Databases; Design parameters; Bored pile design; Humid tropics; Slope stability

1 INTRODUCTION

Geotechnical engineers are frequently called upon to make rapid estimates of soil parameters for use in design and sensitivity studies. Kulhawy and Mayne (1990) presented a variety of databases which were analysed statistically to allow engineers to make a-priori assessments of key foundation design parameters. In this paper the use of databases is discussed for two different scenarios (a) the prediction of the effective friction angles for slope stability modelling in Saint Lucia in the Eastern Caribbean and (b) the prediction of settlement of straight-shafted and under-reamed bored piles in London Clay.

2 SLOPE STABILITY IN THE HUMID TROPICS

By 2050, most urban development and new roads constructed globally will be in non-OECD countries (Dulac 2013) that often have limited resources for geotechnical data collection, analysis and management. Many such countries are in the humid tropics and are prone to rainfall-triggered landslides and this coincides with areas that are ‘data-poor’. The Combined Hydrology And Stability Model, CHASM, has been extensively used for slope stability analysis in such locations (e.g., Anderson et al. 1997, Holcombe et al. 2016, Shepheard et al. 2018a, Beesley et al. 2017). Geodatabases at these locations could augment landslide modelling and sensitivity analysis and help better understand and manage current and future hazards associated with natural and engineered slopes. An available database of effective friction angle measurements on St Lucian soils has been compiled and analysed. Fig. 1 shows a plot of effective peak friction angle plotted against liquidity index. The systematic lack of data is likely contributing to the inherent scatter in the dataset, although multiple-linear regression analysis has helped refine this somewhat (see Shepheard et al. 2018b for more details on the Saint Lucian database analysis).

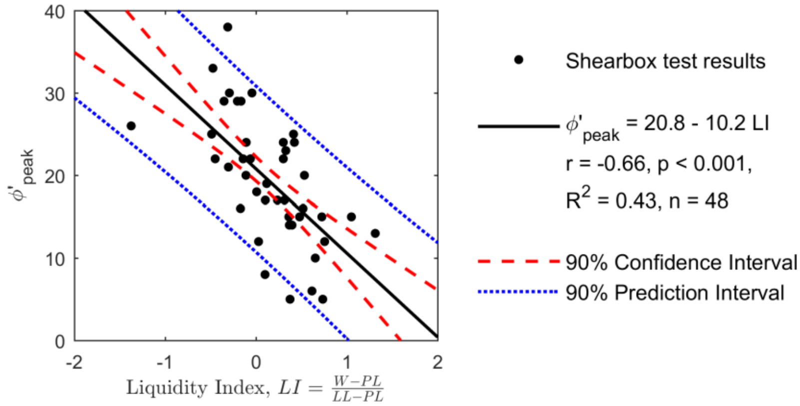

Figure 1. Peak effective friction angle versus liquidity index (data from Shepheard et al. 2018b), where *W* is the water content, *PL* is the plastic limit and *LL* is the liquid limit.

Applying CHASM stochastically (e.g., Almeida et al. 2017) using parameter probability distributions from such a database can allow typical slope stability thresholds and sensitivities to be identified at regional scales, and enable *a-priori* evaluations of design parameters – so long as site specific testing is eventually carried out when engineering design is required.

3 SETTLEMENT OF BORED PILES IN LONDON CLAY

By contrast to the developing world London Clay has been extensively studied, perhaps more than any other geomaterial. In this deposit there has been a history of both well reported pile test data (Skempton 1959, Whitaker and Cooke 1966, Patel 1992) and high-quality laboratory test data (Gasparre et al. 2007). London Clay is highly variable. It can be separated into five depositional sequences corresponding to marine transgression and regression (King 1981). Each subdivision shows an increase in coarser constituent i.e. ranging from a silty clay becoming interbedded clays and sands with increasing distance upwards on each succession (as the subdivisions become younger). Pantelidou and Simpson (2007) describe a good correlation of data within these subdivisions when looking at laboratory classification tests typically used to describe engineering properties. Where a site lies within this stratigraphic sequence will alter the engineering properties encountered, therefore, most site behaviour cannot be accurately modelled without confirmation through site investigation. However, once an engineer is armed with a good understanding of the geological extent and strength variation of the London Clay at a site (e.g., sufficient borehole data and SPTs and/or triaxials) and a model of its stress-strain behaviour, they can make an improved estimate of possible pile settlements.

Vardanega and Bolton (2011a) proposed a simple power model (Eq. 1) to describe soil stress-strain response. This was calibrated using a database of tests on a variety of clays and silts.

$$\frac{1}{M} = \frac{\tau}{c_u} = 0.5\left(\frac{\gamma}{\gamma_{M=2}}\right)^b \quad, 1.25 \leq M \leq 5 \qquad (1)$$

where M is the soil strength mobilisation factor, τ is the shear stress, c_u the undrained shear strength, γ the shear strain, $\gamma_{M=2}$ the shear strain when half the undrained shear strength c_u has been mobilised, and b is an exponent. Vardanega and Bolton (2011b) used a database to compute an average $\gamma_{M=2}$ value of 0.007 and an average b value of 0.58 for M ranging between 1.25 and 5 for London Clay.

Vardanega et al. (2012) developed Eq. (2) for predicting pile head settlement, w_h, at different soil strength mobilisation factors. This model accounts for both the elastic shortening of the pile and the settlement component from the soil mass.

$$\frac{w_h}{D_s} = \frac{b\,\gamma_{M=2}}{2(1-b)}\left(\frac{2}{M}\right)^{1/b} + \frac{2}{M}\frac{\overline{c_u}}{E_p}\left(\frac{L}{D_s}\right)^2 \quad , \; 1.25 \leq \frac{1}{\alpha} \leq M \leq 5 \tag{2}$$

where L is the length of the pile, D_s the pile shaft diameter, E_p the elastic modulus of the pile, $\overline{c_u}$ the average undrained shear strength and α an empirical adhesion coefficient. Eq. (2) is valid when M is greater than $1/\alpha$, at which point the ultimate shaft resistance is reached.

Vardanega et al. (2012) related M to overall factor of safety, F_{total}, using Eq. (3), assuming the factor of safety on the shaft resistance, F_{shaft}, is sufficiently similar to F for straight-shafted piles.

$$F_{total} = \frac{P_{u,s}+P_{u,b}}{P_s+P_b} \approx \frac{P_{u,s}}{P_s} = F_{shaft} = \alpha M \tag{3}$$

where P_s and $P_{u,s}$ are the applied and ultimate shaft loads and P_b and $P_{u,b}$ are the applied and ultimate base loads respectively. However, when $P_{u,b}$, is higher relative to $P_{u,s}$, such as for under-reamed piles, F_{shaft} may deviate from F_{total}. By assuming that no base resistance is mobilised until after the shaft resistance is exhausted ($P_b=0$), the modification proposed in Eq. (4) accounts for this deviation and is more generally applicable to both straight-shafted and under-reamed piles.

$$F_{total} = \frac{P_{u,s}+P_{u,b}}{P_s} = \frac{P_{u,s}+P_{u,b}}{P_{u,s}} F_{shaft} = \frac{P_{u,s}+P_{u,b}}{P_{u,s}} \alpha M \tag{4}$$

$$P_{u,s} = \alpha \overline{c_u} \pi D_s L \;\; ; \quad P_{u,b} = N_c c_{ub} \pi {D_b}^2/4 \tag{5}$$

where c_{ub} is the undrained shear strength at the pile base and N_c is the undrained bearing capacity factor. Substituting Eq. (5) into Eq. (4) yields Eq. (6):

$$F_{total} = \alpha M + \frac{N_c}{4}\frac{c_{ub}}{\overline{c_u}}\frac{D_s}{L}\left(\frac{D_b}{D_s}\right)^2 M \tag{6}$$

The model reported in Vardanega et al. (2012) was found to match the bounds of Patel's (1992) database of test results well, however, it was not applied to individual pile tests in London Clay. In this paper a preliminary investigation of the model's applicability to the full-scale pile load tests reported in Whitaker and Cooke (1966) is presented. Whitaker and Cooke (1966) performed 12 pile load tests and conducted a ground investigation on a site in Wembley, London. For each test pile they conducted a maintained load (ML) test to approximately 70% of estimated capacity (calculated using the α-method: Skempton 1959) then conducted a constant rate of penetration (CRP) test to estimate the final capacity. Pile O had no recorded settlement data and therefore is not discussed further. Discrepancies between the plotted data and tabulated results provided by Whitaker and Cooke (1966) were noted by England (1999). The present authors have opted to use the tabulated results.

Fig. 2(a) depicts the predicted values of ultimate shaft friction τ_u using the α-method (Eq. 7) for both the ML and CRP tests against average shaft friction, $\overline{\tau_u}$, (Eq. 8) during the last load increment of the ML tests and at failure in the CRP test, plotted at half the pile length.

$$\tau_u = \alpha\, c_u \tag{7}$$

$$\overline{\tau_u} = \frac{R_u}{A_s} = \frac{P_u - Q_u}{A_s} \tag{8}$$

where R_u, P_u and Q_u are the shaft resistance, applied load and measured base resistance at 'failure' respectively, approximated as the maximum load applied in the test. The c_u profile was derived by Whitaker and Cooke (1966) from triaxial test data. The chosen α values of α_{ML} = 0.45 and α_{CRP} = 0.60 show good agreement with the test results and are within the ranges reported in Patel (1992).

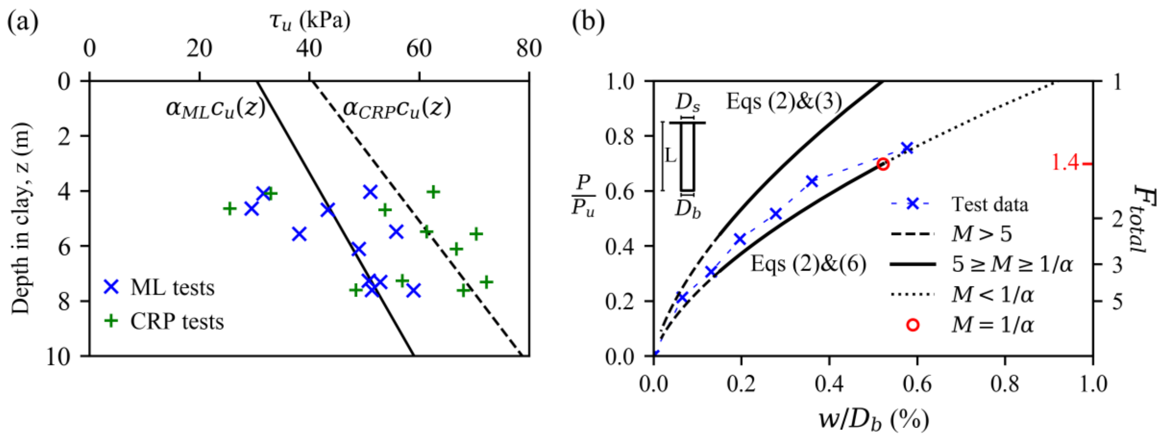

Figure 2. Data from Whitaker and Cooke (1966): (a) predicted shaft friction against depth compared to average measured shaft friction, (b) Normalised load-settlement curve for the ML test on a straight shafted pile (Pile H) compared with predicted curves; L/D_s=15.8, D_b/D_s=1.0, $E_p=20\times10^6$kPa.

Fig 2(b) shows the normalised load-settlement curve for a ML test on a straight-shafted pile (Pile H) compared with two predicted curves. Eqs (2) and (3) underestimate the settlement while Eqs (2) and (6) overestimate it. Eq. (3) assumes that the base resistance mobilises at the same rate as the shaft resistance and Eq. (6) assumes that the base resistance does not mobilise until after the shaft resistance is exhausted. The relatively small difference between the two approaches and the measured data supports the assumption that Eq. (3) is reasonable for straight-shafted piles. The portion of the curves for which the prediction is valid are show using solid lines. The shaft resistance accounts for approximately 70% of the total resistance, therefore the model in Eqs. (2) and (6) is valid for F_{total} greater than approximately 1.4. Fig. 3 shows normalised load-settlement curves for the 10 remaining piles tested by Whitaker and Cooke (1966) compared to the predicted curve from Eqs (2) and (6). The results for the straight-shafted piles (the top four graphs) follow a similar trend to that shown in Fig 2(b). With the modification proposed, the results for under-reamed piles (the bottom six graphs) also show good agreement. However, as the shaft resistance is a lower proportion of the total load, the model can only predict settlements for a smaller range of F_{total}. The average proportion of the total resistance taken by the shaft is 33% for the six under-reamed piles on this site. A model incorporating the response of the pile base would be required to predict settlements at higher applied loads.

4 SUMMARY

Slope stability modelling in the humid tropics can be supported and improved by the assembly, statistical analysis and sharing of existing tropical geotechnical data. Although site specific parameters are difficult to estimate, such datasets could be used for *a-priori* estimates for design and in regional scale modelling. The simple pile-settlement model proposed in Vardanega et al. (2012) has been shown to predict the load-settlement curves for straight-shafted piles from the site at Wembley (London) reported in Whitaker and Cooke (1966) reasonably well. A simple modification has been proposed that allows the method to be applied to under-reamed piles, with promising results when applied to tests from the same site. Additional work should be undertaken to further calibrate this model using an expanded database of pile load tests in key geological deposits.

DATA AVAILABILITY STATEMENT

This research has not generated new experimental data. The authors wish to thank the Government of Saint Lucia Ministry of Infrastructure, Port Services and Transport for supplying us with the Saint Lucia soils data for use in this work (see also Shepheard et al. 2018b for further information).

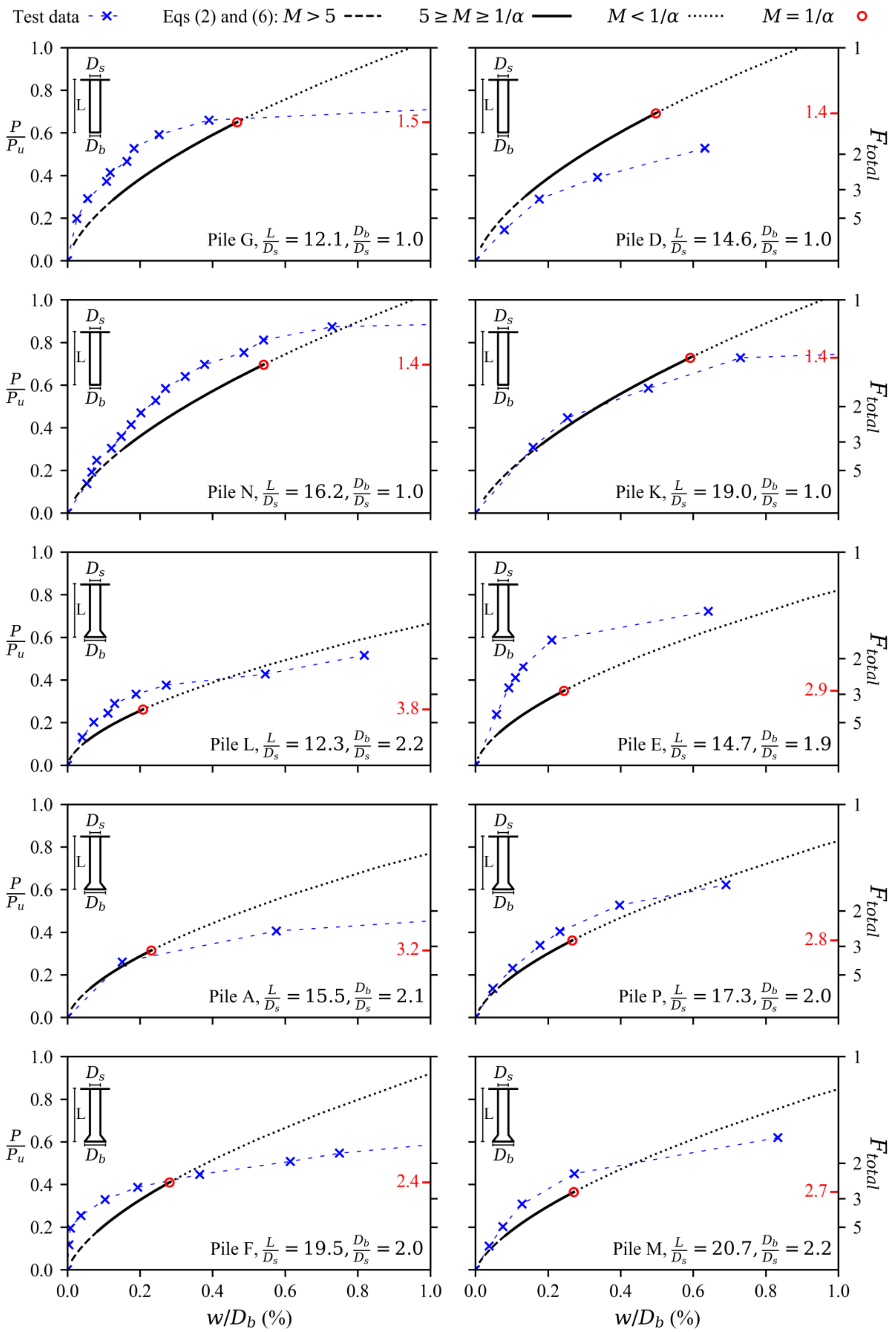

Figure 3. Normalised load-settlement curves for ML tests conducted by Whitaker and Cooke (1966) compared with predicted curve; $\alpha_{ML} = 0.45$, $E_p = 20\times10^6$ kPa.

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